ABSTRACT

The objective of this paper is to assess the pressure capacity of the containment in the ultimate limit state at which the structural integrity is retained. For estimating the pressure capacity of the containment structure by deterministic structural simulation with the aid of computer simulation, the static nonlinear 3D finite element analysis will be needed for predicting the global response. In the case of un-bonded tendons, the possibility of slip between tendon and tendon duct should be enabled by some suitable modelling arrangement like with the use of slot connectors modelling the contact between tendon and grout or grease in the tendon duct. The simplest method to assess the ultimate pressure capacity of the post-tensioned concrete containment vessel (PCCV) is to use analytical formulas. This approach results in pressure-displacement, which gives the mid-height radial displacement in the cylindrical part of the containment plotted against the internal pressure. This curve exhibits five significant corner points, namely: 1) pressure to overcome pre-stress, 2) concrete cracking, 3) liner yield, 4) rebar yield, 5) tendon yield (=ultimate limit state). It will be shown in this paper that the use of analytical formulas the use simple formulas gives satisfactory results for the internal pressure values at these corner point compared to nonlinear finite element simulations or to experimental results.

INTRODUCTION

This document addresses deterministic methods for structural integrity evaluations of containment structures of pre-stressed concrete containments with bonded or un-bonded tendons. Pre-stressed concrete containments include the structural concrete pressure-resisting shells and shell components, steel metallic liners, penetration liners extending the containment liner through the surrounding shell concrete, and tendons and anchorage system.

DESIGN OBJECTIVES AND METHODS FOR BEYOND DESIGN CONDITION

The objective is to assess the pressure capacity of the containment at which the structural integrity is retained, and a failure leading to a significant release of fission products does not occur. For estimating the pressure capacity of the containment structure by deterministic structural simulation with the aid of the static nonlinear 3D finite element analysis will be needed for predicting the global response. Large penetrations are usually included in the finite element model; smaller penetrations and penetration closure components are analyzed using a local finite element models either separate or as parts of modified global finite element model. For cylindrical containment structures the use of analytical solution formulas to estimate the pressure capacity are possible as first link in the analysis sequence. The initial condition for the nonlinear analysis of the containment structure should be the linear elastic response caused by dead load and design pressure, at the
design temperature. The internal pressure is incrementally increased until a failure is reached. The nonlinear stress-strain curve for steel materials should be based on the code-specified minimum yield strength for the specific grade of steel and a stress-strain relationship beyond yield that is representative of the specific grade of steel in the relevant temperature conditions. The nonlinear stress-strain curve for concrete should be based on experimentally verified test results corresponding to relevant temperature conditions. The pressure capacity for cylindrical pre-stressed concrete containments is to be based on the contribution from each structural element considered in the analysis, using the stress-strain curve for each material and the strain level in each material, as determined based on overall strain compatibility between all of the credited structural elements such as concrete, liner, tendons and reinforcement bars. In addition for estimating the cylindrical shell capacity in ideal membrane state, the analysis should consider additional failure modes, such as concrete shear and concrete crushing which may occur near discontinuities. The analysis methods described above apply to the global containment capacity. A complete evaluation of the internal pressure capacity should also address major local components such as equipment hatches, personnel airlocks, and major piping penetrations. The possibility of slip between tendon and tendon duct should be enabled by some suitable modelling arrangement like with the use of slot connectors modelling the contact between tendon and grout or grease in the tendon duct. This means that the tendons as general rule should be modelled explicitly using bar elements in the containment model. The preferable arrangement of the meshing of the composite model consisting of tendons and containment wall concrete is the coincident nodes of tendon mesh consisting of rod elements and concrete mesh consisting of shell or solid elements. Regulating the connector stiffness of slot connector either perfect or total lack of bond between tendon and concrete can be simulated. Cracking of concrete can be simulated using various model approaches depending on the precision and size of the model. So called smeared crack concept is one modelling possibility assuming that decreased stiffness caused by concrete cracking is distributed evenly on the tributary area of material connected to one integration point inside one particular finite element. Other possible modelling approaches could be the brittle cracking modelling approach or damaged plasticity modelling approach or individual micro-plane modelling approach specifying the one dimensional stress-strain relationship of the concrete material for a multitude of directions called micro-planes.

ILLUSTRATIVE EXAMPLES FOR ESTIMATING THE ULTIMATE PRESSURE CAPACITY AND BEYOND DESIGN LOAD BEHAVIOR OF CONTAINMENTS, REF. [1]

Sandia scale model

The following definitions refer to the formulae in this subsection to use analytical formulas to develop the pressure capacity of 1:4-Scale Pre-stressed Concrete Containment Vessel Model tested to ultimate failure in Sandia laboratories. The tendons in Sandia containment model specimen were un-bonded.

**Prestress load and corresponding radial displacement**

\[ p = \text{reinforcement ratio} = 0.0228 \]

\[ \rho_{\text{hoop rebars}} = \rho_{\text{hr}} = \frac{\text{Area of hoop reinforcement/gross concrete area}}{0.00865} \]

\[ \rho_{\text{liner}} = \frac{\text{Area of liner/gross concrete area}}{0.00492} \]

\[ \rho_{\text{hoop tendons}} = \rho_{\text{ht}} = 0.00927 \]

\[ t_{\text{liner}} = \text{thickness of liner} = 0.16 \text{ cm} \]
\( t_{eq} \) = equivalent concrete thickness or transformed section thickness (concrete section area)
With steel portion transformed by ratio of Young's Moduli = 35.2 cm
\( t'_{eq} \) = \( t_{eq} \) including rebar and tendons 37.0 cm
\( t_c \) = thickness of concrete wall = 32.5 cm
\( \sigma_{o\text{(concrete)}} \) = compressive concrete stress after prestressing = -8.83 MPa
\( R \) = Inside radius of cylinder = 538 cm
\( E_{\text{rebar}}, E_{\text{tendon}}, E_{\text{liner}} \) = Young’s Moduli of rebar, tendon, and liner = 200 000 MPa
\( E_c \) = Young's Modulus of concrete = 33 000 MPa
\( \varepsilon_{cr} \) = Concrete cracking strain = 80 \times 10^{-6}
\( \varepsilon_{ly} \) = liner yield strain = 270/200 000 = 0.00135
\( \varepsilon_{ry} \) = rebar yield strain = 470/200 000 = 0.00235
\( \sigma_{\text{linerult}} \) = liner ultimate strength = 498 MPa
\( \sigma_{\text{barult}} \) = rebar ultimate strength = 658 MPa
\( \sigma_{\text{tendonult}} \) = tendon ultimate strength = 1876 MPa

**Pre-stress load and corresponding radial displacement**

There are four hoop tendons of area 3.39 cm² in every 45 cm wall segment.
\( \rho_{\text{hoop,tendons}} = (4 \times 3.39 \text{ cm}^2) / (32.5 \times 45) = 0.00927 \)

In compression under tendon action,
\( \sigma_0(\text{concrete}) = -\rho_{\text{tendon}}\sigma_{\text{tendon}} = -0.00927 \times 953 \text{ MPa} \)
(Average prestress in hoop tendons including assumed losses).
\( \sigma_0 = -8.83 \text{ MPa} \)

Inward pressure \( P_{\text{inward,prestress}} \) to correspond prestress, \( F_{\text{prestress}} \) is
\( P_{\text{inward,prestress}} = \sigma_0 * t_{eq} / R = -8.83 \times 35.3 / 538 = -0.580 \text{ MPa} \)

Corresponding inward radial displacement = (-8.83) / (33000) * 538 * 10 mm ≈ -1.44 mm

**Pressure at which cylinder stress overcomes pre-stress, \( P_o \)**

In compression under tendon action,
\( \sigma_0(\text{concrete}) = -\rho_{\text{tendon}}\sigma_{\text{tendon}} = -0.00927 \times 953 \text{ MPa} \)
(Average prestress in hoop tendons including assumed losses)
\( \sigma_0 = -8.83 \text{ MPa} \)

Pressure to overcome prestress, \( P_o \) is
\( P_o = \sigma_0 * t_{eq} / R = 8.83 \times 35.3 / 538 = 0.580 \text{ MPa} \)

Corresponding radial displacement = 0 cm

**Cylinder hoop cracking pressure, \( P_{hc} \)**

\( P_{hc} = t_c * E_c * \varepsilon_{cr} / R + P_o \approx 32.5 \times 33000 \times 80 \times 10^{-6} / 538 + 8.83 \times 35.3 / 538 = 0.74 \text{ MPa} \)

Radial displacement at concrete cracking = 0.00008 * 538 * 10 mm ≈ 0.43 mm

**Pressure at Liner Yield, Ply**

Assuming the tendons have not yielded, the hoop stiffness after cracking is approximately that of elastic rebar, liner and tendons acting alone. Therefore:
\( \varepsilon_{ly} / E_{\text{steel}} = 270 / 200000 = 0.00135 \)

Solving,
\( P_l = (0.00135 \times 0.0228 \times 32.5 \times 200000) / 538 + 8.83 \times 35.3 / 538 = 0.95 \text{ MPa} \)

Liner yield displacement = 0.001 * 538 * 10 mm ≈ 5.38 mm
(To calculate the liner yield displacement the steel strain is assumed to be 35% higher than the average shell strain, which means that the average shell strain calculated for whole circumference is less than the steel strain at cracks because of the stiffness increase in un-cracked portions of shell wall).

**Pressure at rebar yield, P_{ry}**

Assuming the tendons have not yielded, the hoop stiffness after cracking is the stiffness of elastic rebar, liner and tendons acting alone. Therefore,

\[ \varepsilon_{ry} = \frac{\sigma_{ry}}{E_{steel}} = \frac{470}{200000} = 0.00235 \]

Solving,

\[ P_{ry} = \frac{(0.00235 \times (0.0228 - 0.00492) \times 32.5 \times 200000)}{538} + \frac{(0.00135 \times 0.00492 \times 32.5 \times 200000)}{538} + 8.83 \times 35.3 / 538 = 1.17 \text{ MPa} \]

Rebar yield displacement \( d_{rebar,yield} \approx 0.002 \times 538 \times 10 \text{ mm} \approx 10.76 \text{ mm} \)

(To calculate the rebar yield displacement the steel strain is assumed to be 35% higher than the average shell strain, which means that the average shell strain calculated for whole circumference is less than the steel strain at cracks because of the stiffness increase in un-cracked portions of shell wall).

**Ultimate cylinder membrane failure based on ultimate strengths of steel components, P_{ult}**

\[ P_{ult} = \frac{(0.006 \times (0.0228 - 0.00865) \times 32.5 \times 200000)}{538} + \frac{(0.00235 \times 0.00865 \times 32.5 \times 200000)}{538} + \frac{(0.00135 \times 0.00492 \times 32.5 \times 200000)}{538} + 8.83 \times 35.3 / 538 = 1.57 \text{ MPa} \approx 4 \times P_d \]

Ultimate radial displacement, \( d_{ultimate} \approx 0.006 \times 538 \times 10 \approx 32.3 \text{ mm} \)

The developments calculated above are presented in Table 1 and in plot format in Ошибка! Источник ссылки не найден. 1 and Ошибка! Источник ссылки не найден. 2 below.

<table>
<thead>
<tr>
<th>1:4 Sandia containment model test</th>
<th>Pressure [MPa]</th>
<th>Rad disp. [mm]</th>
<th>Corresponding pressure at test</th>
<th>Corresponding displacement at test</th>
</tr>
</thead>
<tbody>
<tr>
<td>prestress</td>
<td>0</td>
<td>-1.44</td>
<td>0 (green curve)</td>
<td>-1.78</td>
</tr>
<tr>
<td>pressure to overcome prestress</td>
<td>0.58</td>
<td>0</td>
<td>0.582 (green curve)</td>
<td>0</td>
</tr>
<tr>
<td>concrete cracking</td>
<td>0.74</td>
<td>0.43</td>
<td>0.595 (green curve)</td>
<td>0.662</td>
</tr>
<tr>
<td>liner yield</td>
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<td>5.38</td>
<td>0.972 (green curve)</td>
<td>8.13</td>
</tr>
<tr>
<td>rebar yield</td>
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<td>10.76</td>
<td>1.111 (green curve)</td>
<td>11.7</td>
</tr>
<tr>
<td>tendon yield</td>
<td>1.57</td>
<td>32.3</td>
<td>1.400 (red curve)</td>
<td>79.2</td>
</tr>
</tbody>
</table>

Table 1 Pressure – displacement relationship showing the conceptual sequence developed by using analytical formulas for the cylinder of the infinite length of Sandia 1:4 model test up to the pressure capacity.

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Figure 1 Conceptual corner points of Sandia 1:4 containment model test, the design pressure for the Sandia containment model was 0.39 MPa.

Figure 2 The actual pressure – radial displacement relationship in the Sandia 1:4 containment model test.
The green curve in Figure 2 shows pressure – radial displacement relationship in cylinder mid-height for leak-rate (LST) test performed for un-cracked containment and the red curve shows radial displacement relationship in cylinder mid-height for ultimate pressure capacity test (SFMT) performed for already cracked containment. The yellow curve (simple anal. disp.) repeats the conceptual corner points of the curve in Figure 1. The blue curve (SOL 5 unbonded) shows the FEM simulation results of the Sandia test specimen with un-bonded tendons. The brown curve (SOL 5 bonded) shows the FEM simulation results of the Sandia test specimen with un-bonded tendons.

It can be seen from Figure 2 that estimated ultimate capacity in tendon modelling with bond is 1.56 MPa and that estimated ultimate capacity in tendon modelling without bond is 1.44 MPa meaning 8% difference in estimated ultimate capacity.

OL3 EPR containment, 2005

The tendons in OL3 containment design are bonded.

**Prestress load and corresponding radial displacement**

There is one hoop tendon of area $154 \mathrm{cm}^2$ in every meter wall segment with wall thickness of 1.3 meters.

\[
\rho_{\text{hoop tendons}} = \frac{187 \mathrm{cm}^2}{130 \times 100 \mathrm{cm}^2} = 0.01438
\]

In compression under tendon action,

\[
\sigma_0(\text{concrete}) = -\rho_{\text{tendon}}\sigma_{\text{tendon}} = -0.01438 \times 960 \text{ MPa, (Avg. prestress in hoop tendons including assumed losses)}
\]

\[
\sigma_0 = -13.8 \text{ MPa}
\]

Inward pressure $P_{\text{inward prestress}}$ to correspond prestress, $F_{\text{prestress}}$

\[
P_{\text{inward prestress}} = \sigma_0 \times t / R = -13.80 \times 1300 / 24050 = -0.746 \text{ MPa}
\]

Corresponding inward radial displacement $= (-13.80) / (33500) \approx -9.91 \text{ mm}$

**Pressure at which cylinder stress overcomes prestress, $P_o$**

In compression under tendon action,

\[
\sigma_0(\text{concrete}) = -\rho_{\text{tendon}}\sigma_{\text{tendon}} = -0.01438 \times 960 \text{ MPa (Average prestress in hoop tendons including assumed losses)}
\]

\[
\sigma_0 = -13.8 \text{ MPa}
\]

Pressure to overcome prestress, $P_o$, is

\[
P_0 = \sigma_0 \times t / R = 13.8 \times 1300 / 24050 = 0.746 \text{ MPa}
\]

Corresponding radial displacement $= 0 \text{ cm}$

**Cylinder hoop cracking pressure, $P_{hc}$**

\[
P_{hc} = t_c \times E_c \times \varepsilon_{cr} / R + P_0 \approx 1300 \times 33500 \times 100 \times 10^{-6} / 24050 + 13.8 \times 1300 / 24050 = 0.93 \text{ MPa}
\]

Radial displacement at concrete cracking $\approx 0.0001 \times 24050 \approx 2.4 \text{ mm}$

**Pressure at liner yield, $P_{ly}$**

Assuming the tendons have not yielded, the hoop stiffness after cracking is approximately that of elastic rebar, liner and tendons acting alone. Therefore:

\[
\varepsilon_{ly} = \frac{\sigma_{ly}}{E_{\text{steel}}} = 270 / 200000 = 0.00135
\]

\[
p = \text{reinforcement ratio} = 0.004615 + 0.004615 + 0.01438 = 0.02361
\]

\[
\rho_{\text{hoop rebar}} = \rho_{hr} = \text{Area of hoop reinforcement/gross concrete area} = 60 / 130 / 100 = 0.04615
\]

\[
\rho_{\text{liner}} = \text{Area of liner/gross concrete area} = 6 / 1300 = 0.004615
\]

\[
\rho_{\text{hoop tendons}} = (187 \mathrm{cm}^2) / (130 \times 100 \mathrm{cm}^2) = 0.01438
\]

Solving,

\[
P_{ly} = (0.00135 \times 0.02361 \times 1300 \times 200000) / 24050 + 13.8 \times 1300 / 24050 = 0.775 \text{ MPa}
\]

Liner yield displacement $\approx 0.00075 \times 24050 \approx 18.04 \text{ mm}$

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(To calculate the liner yield displacement the steel strain is assumed to be 40% higher than the average shell strain, which means that the average shell strain calculated for whole circumference is less than the steel strain at cracks because of the stiffness increase in un-cracked portions of shell wall).

**Pressure at rebar yield, \( P_{ry} \)**

Assuming the tendons have not yielded, the hoop stiffness after cracking is approximately that of elastic rebar, liner and tendons acting alone. Therefore:

\[
\varepsilon_{ry} = \frac{\sigma_{ry}}{E_{steel} = 470 / 200000} = 0.00235
\]

\[
P_{ry} = \frac{(0.00235 \times (0.02361 - 0.004615) \times 1300 \times 200000)}{224050 + 13.8 \times 1300 / 24050} = 1.17 \text{ MPa}
\]

Rebar yield displacement \( d_{rebar, yield} \approx 0.002 \times 24050 \approx 48.1 \text{ mm} \)

(To calculate the rebar yield displacement the steel strain is assumed to be 35% higher than the average shell strain, which means that the average shell strain calculated for whole circumference is less than the steel strain at cracks because of the stiffness increase in un-cracked portions of shell wall).

**Ultimate cylinder membrane failure based on ultimate strengths of steel components, \( P_{ult} \)**

\[
P_{ult} = \frac{(0.006 \times (0.02361 - 0.004615 - 0.004615) \times 1300 \times 200000)}{24050 + (0.00235 \times 0.004615 \times 1300 \times 200000)} / 24050 + 13.8 \times 1300 / 24050 = 1.86 \text{ MPa} \approx 4.13 \times P_d
\]

The conceptual corner points of Olkiluoto3 containment design are given Figure 3.

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**Figure 3**  Conceptual corner points of Olkiluoto3 containment design, the design pressure for OL3 containment was \( P_d = 0.45 \text{ MPa} \).

**ULTIMATE CAPACITY BY 3D MODELLING**

In the following Figure 4 the 3D model of the Sandia scale model is given.
Figure 4  3-D finite element mesh of the Sandia containment scale model

In the following Figure 5 the 3D model of the OL3 containment is given.

Figure 5  the 3D model of the OL3 containment
CONCLUSION

This document described briefly the beyond loading situations of the nuclear containments and the methods to assess the ultimate pressure capacity of nuclear containments. Geometric and material modelling were presented as well as the obtained results for the ultimate pressure capacity of the nuclear containments with the aid of simple hand calculations and with the aid of sophisticated 3D finite element analyzes.

REFERENCES